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DISCUSSION OF PROCEEDINGS PAPERS

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Discussion of
"STABILITY OF BEAM COLUMNS ABOVE THE ELASTIC LIMIT"

By R. L. Ketter, ASCE
(Proc. Paper 692)

A. HUBER.*—In the discussion of existing solutions to the beam-column problem the author points out that such solutions have been based mainly on the criterion of instability that involves the maximum point of a load-deflection curve. However, the basic equation formulating a new instability criterion was derived by the author by virtual displacement considerations.

Subsequently it will be shown that the former criterion of instability can be used to arrive at the same basic equation given by the author (Eq. 5).

Considering the symmetric beam-column shown in Figure 5, the condition of equilibrium at the center of the column can be written in the author's notation as follows:

$$M_o = M_e + P \cdot y_o + R \cdot a \quad - - - - - \text{(a)}$$

where a is the distance of the lateral forces R from the pin ends.

Considering P and $M_e = P \cdot e$ as primary variables, the maximum value of P must be found for a given column length, L, eccentricity, e, and lateral force, R. Expressing P explicitly in Eq. (a) the following equation results:

$$P = \frac{M_o - R \cdot a}{y_o + e} \quad - - - - - - - - - - \text{(b)}$$

If P were given as function of Y_o, the maximum value of P would be obtained from

$$\frac{dP}{dy_o} = \frac{d}{dy_o} P\left[\varphi(y_o)\right] = \frac{\partial P}{\partial \varphi}\frac{d\varphi}{dy_o} = 0$$

Proceeding with a formal differentiation of Eq. (b) we obtain

$$\frac{\partial P}{\partial \varphi}\frac{\partial \varphi}{\partial y_o} = \frac{\frac{\partial M_o}{\partial \varphi}\frac{\partial \varphi}{\partial y_o}(y_o + e) - M_o}{(y_o + e)^2} + \frac{R \cdot a}{(y_o + e)^2} = 0$$

Substituting from Eq. (a) $R \cdot a - M_o = -P \cdot (e + y_o)$ and simplifying we get

$$\frac{\partial M_o}{\partial \varphi}\frac{\partial \varphi}{\partial y_o} - P = 0$$

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or

$$P_{cr} = \frac{\partial M_o}{\partial \varphi} \frac{\partial \varphi}{\partial y_o} \quad \text{- - - - - - - - - - - - (c)}$$

where φ can be any convenient parameter formulating the problem, for instance the curvature ϕ_0. Then

$$\frac{\partial M_o}{\partial \phi_o} = P_{cr} \frac{\partial y_o}{\partial \phi_o}$$

and

$$\left.\frac{\partial M}{\partial \phi_o}\right|_{ext.} = \frac{\partial}{\partial y_o} \left[P_{cr}(e + y_o) \right] \frac{\partial y_o}{\partial \phi_o} = P_{cr} \frac{\partial y_o}{\partial \phi_o}$$

or

$$\left.\frac{\partial M}{\partial \phi}\right|_{ext.} = \left.\frac{\partial M}{\partial \phi}\right|_{int.} \quad \text{- - - - - - - - (d)}$$

Eq. (d) is identical with the author's Eq. (5), which is the new instability criterion.

The real contribution of the paper is not only in the presentation of the new criterion of instability but also in the method of actual evaluation and application of the criterion to a variety of problems. The usefulness and power of the method is thereby clearly demonstrated.

Discussion of
"ANALYSIS FOR SHEET PILE RETAINING WALLS"

by F. E. Richart
(Proc. Paper 694)

P. W. ROWE.*—Mr. Richart has made an interesting attempt to produce an analytical design procedure for sheet-pile walls which takes into account the relative flexibility of the pile and the soil. His main additional objects have been

1. To provide for varying strata above the dredge level
2. To present the design data in a form in which it can be used by a practising civil engineer.

The first objective was based on the belief that the writer's experimental tests[3] were made with sand at the same density above and below the dredge level. However, Figs. 16(a) and (b) of Ref. 3 show that tests were made with dense sand below the dredge level and loose sand above, and in the Correspondence[13] following the report of those tests, further results were given for the case of water pressure above loose and dense sand. Both these tests and mathematical analyses[14] of the problem show that it is only the stiffness of the subsoil below the dredge level which reduces the moment below that for an infinitely stiff wall. This latter value is calculated by the Free Earth Support Analysis which takes into account the varying strata, differential water pressures etc. Thus in Figs. 21, Ref. 3, the ordinates of the Operating Curves vary from 17 to 4 for very poor to very good site conditions.

Mr. Richart then states that "considerable judgment is required in order to extrapolate this test information for use in actual pile design." Considerable judgment is of course always necessary in civil engineering, but the writer should be grateful if Mr. Richart could explain this statement further.

The second objective, in the writer's opinion, has not been reached. Firstly the design has to proceed indirectly by trial and error. A section must be chosen and the stress checked. To choose the section in the first instance, the "equivalent beam" procedure is used. This means that two sets of calculations are required. In addition, the process of checking, involves extrapolation from eight graphs and many tabulations.

In January 1955 the writer published a theoretical and experimental study of sheet pile walls, pinned[14] and encastre,[15] at the anchorage and Mr. Richart's basic differential equation is identical with the one which the writer

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13. Correspondence on Ref. 3.
Proc. Inst. Civ. Engrs. London. Part I, Sept. 1952.

14. P. W. Rowe. "A Theoretical and Experimental Analysis of Sheet-Pile Walls."
Proc. Inst. Civ. Engrs. London. Part I, vol. 4, p. 32. (Jan. 1955).

15. P. W. Rowe. "Sheet Pile Walls Encastre at the Anchorage."
Pt. I, vol. 4, p. 70. (Jan. 1955).

developed from stress strain theory of sand. However, the writer completed the analysis of 135 theoretical structures and expressed the maximum bending moments as a ratio of the corresponding values for an infinitely stiff wall subject to the same strata conditions. As a result he found that, in agreement with previous experimental work,[3] the moment reduction depended mainly on the relative stiffness of pile and subsoil, and to a lesser degree on the penetration depth. The amount of surcharge, position of tie rod, or varying densities above the dredge level had no influence on the percentage reduction. Consequently, previously published data was extended to the whole range of compressible cohesionless subsoils while the simplicity of using a single reduction curve on the Free Earth Support value for direct design was retained.

While the writer agrees with the form of equation (4) he cannot follow the reasons for the derivation of equation (3). This also applies to equation (19).

The value C is identical with m in the writer's papers. Now the value of m decreases with soil strain. Consequently the value of m increases from a low value at the dredge level to a maximum at the pile toe, for flexible piling. Such analyses as Richart's and the writer's assume that m is constant, and since very large variations in m are necessary to cause a significant variation in pile fixity, this procedure is admissible. However, it is not clear why it is satisfactory to use the value of Cn equ. 19, at the bottom of the pile.

The penetration depths of piling commonly range from 6 to 18 ft. so that the mean overburden pressures at one half the penetration depth in submerged sand on the "passive" side range from 200 to 600 lb. per sq. ft. At these values the stress strain curves for sand are steeper than at the pressures quoted. In addition the important stiffness of the subsoil is that close to the dredge level, where the confining pressures are very small.

For these reasons the value of C may be higher than those given. In Table III the values of C given in columns 5 and 6 of Table I are compared with the m values previously published. For this purpose the units of C have been changed to lb./ft.^{3} and the logarithms are given.

TABLE III

	log C col 5	log C_{10} col 6	log m
loose sand	4.32	4.60	4.75
Disturbed	4.86	5.17	
Dense	5.57	5.88	6.10

Quite reasonable agreement is reached between log m and log C_{10}. However, it would appear that C_{10} increases with increase in scale, whereas m tends to decrease with increase in scale as would be expected from the decrease of the slope of the ϕ - strain curve for sands with confining pressure.

The writer has called m the "soil stiffness modulus." Since m or C increases in value with increase in stiffness, the value C does represent stiffness rather than compressibility. In addition the word "coefficient" indicates a dimensionless number and the whole term can be confused with the "coefficient of compressibility" used by Terzaghi in confined compression tests. Therefore the writer suggests that the term "soil stiffness modulus" is preferable.

In spite of these criticisms the writer congratulates Mr. Richart on his interesting studies.

Discussion of
"BLAST RESISTANT BUILDING FRAMES"

by Bruce C. Johnston and Archie Mathews
(Proc. Paper 695)

KEITH E. McKEE,[1] J.M. ASCE.—Blast resistant design is currently a problem of interest to many people. At the present time the literature on this subject is either classified or widely scattered, resulting in little available information for the structural designer. The authors' paper is valuable in that it introduces the subject and focuses attention on the problem. The writer feels, however, that in striving for simplicity the authors present an incomplete picture of the problem.

The class of buildings and type of loadings considered by the authors are extremely limited. This by no means detracts from the value of the paper, but does limit its applicability. It is hoped that by discussing some of the limitations, the complexity of the problem of designing a structure to resist blast can be demonstrated. All of these limitations were mentioned in more or less detail by the authors. It is the writer's opinion, however, that the problem was presented as being much more readily solved than is the case for the vast majority of structures. The limitations discussed in the following paragraphs are not intended as an exhaustive list.

The method presented is applicable only to buildings in which the wall covering can be neglected in computing loads. The authors also mention the design of completely closed structures with walls that do not fail. (Ref. 5) For many buildings either architectural or utilitarian requirements prevent the use of either of these extremes in design. The majority of all structures that have been and will be built are 'partially open,' i.e., contain sections of the walls which are 'frangible' and other sections which will fail only under very high loads. A masonry wall with window openings is a typical construction which might be considered 'partially open.' To the best of the writer's knowledge there is no information available in the general literature concerning the loading on 'partially open' buildings.

In this paper, the blast wave is assumed to travel in a direction more-or-less parallel to the plane of the bents. In some cases, for example at the extremities of a small city, the designer might anticipate where the bomb would be dropped and orient the building to satisfy this condition if he so desires. In general, however, it would be necessary to make provisions for blast forces in any direction. The methods of increasing blast resistance in a direction perpendicular to the bents are in many respects different from those in the plane of the bents. If the orientation of the structure is that considered in this paper, the lateral resistance could be substantially increased by placing shear walls in the plane of the bents. Such walls could be of masonry or concrete construction and might serve in addition as fire walls.

The drag loads considered in this paper were based on a drag coefficient

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of approximately two, which applies to members which are more or less rectangular in cross section, including I and WF structural shapes. If it were possible to use members of circular cross section, the drag coefficient would be only 0.35.[2] If one-half of the area exposed to drag forces could be constructed of members having circular sections, the average pressure over the structure could be reduced by over 40 percent. The possible use of such members could be dependent, of course, on the structural details and the overall economy.

In discussing the effect of the positive phase duration of the blast wave on deflections the authors determined that for a building designed so that the lateral resistance exactly equalled the expected peak horizontal force, the ratio of maximum to yield deflection would be between 0.8 and 10. This analysis neglected the effect of dead loads. It was stated that the latter deflection would represent severe permanent deflections short of collapse. This is true for structures similar to those used in the design problem, but can be false for other types of one story steel industrial buildings. For tall columns using light sections and designed primarily for dead loads, as would be the case of a concrete slab roof is used, ten yield deflections may well be associated with static collapse, i.e., will collapse due solely to the effective moment of the dead weight. For example for 20 foot high 8WF31 columns (perhaps the most common size) fixed at the base, the yield deflection would vary from 3.75 to 7.5 inches depending on the comparative bending resistance of the columns and the roof construction. In the strong direction[3] dead loads of only 40 to 20 percent of the design axial load would be required to collapse the structure at 10 yield deflections. This situation will similarly occur for many light sections in the strong direction and some intermediate and light sections in the weak direction.

The above discussion demonstrates that the duration of the blast wave may not be negligible for all types of steel construction. It also demonstrates the necessity of checking for static collapse in any problem involving large deflections of the structure. In connection with this discussion of deflections, it might also be questioned if rigging techniques are sufficient to right any but the most idealized buildings. For many buildings the deflections would be expected to vary from bent to bent and column to column due to nonuniformity of members and loading. Under such circumstances, righting of the structure even if it does not collapse would appear to be an extremely involved if not impossible task.

Design for lateral forces, as recommended in this paper is applicable not only to steel structures, but equally well to concrete or other constructions. Only the supplementary information, such as the calculation of plastic moments, etc., applies solely to steel construction. Since the required supplementary data for other types of construction is normally available to the designer, this presents no serious limitations.

In discussing the plastic moments of steel beams, the authors refer to increases of from 10 to 20 percent over the elastic bending capacity. This increase is obtained only in the strong direction of the member. In the weak

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2. O. Flachsbart, "Modell versuche uber die Belastung, von Gitterfachwerken durch Windkrafte," Der Stahlbau, Apr. 1934, p. 65.
3. Strong and weak directions are used here to refer to the direction in which the column is bent. The strong direction refers to bending about an axis perpendicular to the web, i.e., axis X-X in the AISC Manual. The weak direction refers to bending about the web, i.e. the axis Y-Y in the AISC Manual.

direction, the increase is approximately 50 percent.

In discussing dynamic analysis the tacit assumption is made that the structure can be treated as a single degree of freedom system. In addition to limiting consideration to one-story buildings, this implies a number of other assumption regarding structural action. Among these is the 'rigid roof,' the movements of which can be defined by a single parameter. This requires that no individual components fail under the blast loading. For buildings which are long in the direction of flow, even elastic deflection of the roof may be sufficient to invalidate the assumption of a rigid roof.

The suggestion of using a flat reinforced concrete roof for the type of design considered in this paper is highly desirable. This type of roof construction is probably the most nearly 'rigid roof' which is possible. In addition this type of roof construction adds strength against lateral forces, reduces the load transmitted into the frame, and has greater resistance to direct blast damage. Some of the advantages listed by the authors to justify the use of flat reinforced concrete roofs, however, are at best doubtful.

Among these doubtful justifications is the advantage of the increased mass in reducing the acceleration. Reduced acceleration is not always desirable per se since it is not necessarily associated with reduced maximum displacements under all types of loading. Even if the maximum displacement is reduced, the contribution of the added weight to overturning may more than offset this advantage. Assuming that the column size remains unaltered, the overturning moment is proportional to the dead weight for the same deflection. To reduce the maximum overturning moment, which may be important in causing static collapse, it would be necessary that the maximum displacement be reduced more rapidly than the weight is increased. Similarly, the column size would be increased directly to achieve the desired lateral resistance, without resorting to the indirect method of using a reinforced concrete slab to increase the dead weight and thereby the column size.

The authors say that the goal of blast resistant design "should be to provide the maximum possible increase in lateral resistance to deflection for the minimum added cost." This general advice is equally applicable to any type of structure subjected to blast loading. By striving toward this goal the designer can do much to increase the blast resistance of a structure with little additional effort. Generally speaking, the higher the static load for which the structure is designed, the greater will be its blast resistance. The actual static load to be used in design would be dependent on the additional funds which were available for increasing the static resistance. It would be up to the designer in each situation to arrive at a compromise between blast resistance and additional cost.

Design for a static load may be a satisfactory means of increasing the blast resistance, but the designer should realize that this is at best an approximation which can introduce serious errors. To determine the action of a structure more accurately under a given load, it is necessary to carry out a dynamic analysis taking into account insofar as possible all the complicating factors. To recommend that the design for lateral resistance be based on a static pressure (1400 psf) determined for a specific bomb size, height of burst, and ground range is misleading for two reasons. First, the designer may be interested in a structure under other conditions, either more or less severe. The value selected by the authors is in no way significant. Second, the association of a static design load with bomb parameters is not possible in general. To determine the bomb parameters associated with a lateral design load, the writer feels that a dynamic analysis is mandatory.

The authors acknowledged the importance of dynamic analysis even for the simplest case when they check to find the maximum deflection of their design under the assumed loading. For this reason it is difficult to understand the lack of importance which the paper attaches to the need for such analyses. For simple structures it is not necessary to carry out a numerical solution to determine the full deflection-time history as is done in the Appendix. Solutions to these problems are available in the form of charts so that the ratio of maximum to yield deflection can be determined. (Reference 6 is one source of such charts.) Without a dynamic analysis the designer knows little except that he has increased the blast resistance by increasing the lateral resistance.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

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c. Discussion of several papers, grouped by Divisions.